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Geomembranes for repair of concrete hydraulic structures

by
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The Corps of Engineers operates and maintains a wide variety of hydraulic structures, including mass concrete gravity dams, rockfill dams with concrete facings, roller-compacted concrete dams, navigation locks, flood walls, and concrete-lined river channels. Concrete appurtenances associated with such hydraulic structures include intake towers, outlet works, and guidewalls. Located at over 600 project sites throughout the United States, these structures are subjected to a wide spectrum of environmental conditions. Also, the advanced ages of these structures, more than 40 percent of which are over 50 years old, increases the potential for concrete deterioration.

Many of these structures exhibit significant concrete cracking which allows water intrusion into or through the structure. These cracks are the result of a variety of phenomena, including restrained concrete shrinkage, thermal gradients, cycles of freezing and thawing, alkali-aggregate reaction, and differential settlement of the foundation. Water leakage through hydraulic structures can also result from poor concrete consolidation during construction, improperly prepared lift or construction joints, and waterstop failures.

When leakage rates become unacceptable, repairs are made. Conventional repair methods generally consist of localized sealing of cracks and defective joints by cementitious and chemical grouting, epoxy injection, or surface treatments. Even though localized sealing of leaking cracks and defective joints with conventional methods has been successful in some applications, in many cases some type of overall repair is still required after a few years. Consequently, the potential for geomembranes in such repairs is being evaluated as part of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program.

Background

Various configurations of geomembranes have been used as impervious synthetic barriers in dams, particularly in Europe, for more than 30 years. Generally, membranes are either placed within an embankment or rockfill dam as part of the impervious core or at the upstream face of embankment, rockfill, and concrete gravity dams. In recent years, geomembranes have been increasingly used for seepage control in a variety of civil engineering structures, including canals, reservoirs, storage basins, dams, and tunnels. Geomembranes have also been used successfully to resurface the upstream face of a number of old concrete and masonry dams, particularly in Europe.

Definition

Consistent with the International Commission on Large Dams (ICOLD 1991) the term "geomembrane" is used herein for polymeric membranes which constitute a flexible, watertight material with a thickness of one-half to a few millimeters. A wide range of polymers, including plastics, elastomers and blends of polymers are used to manufacture geomembranes.

Fabrication

Since the existing concrete or masonry surfaces to receive the geomembrane are usually rough, a geotextile is often used in conjunction with the geomembrane to provide protection against puncturing. For example, the geocomposite SIBELON CNT consists of a nonwoven, needle-punched polyester geotextile which is bound to one side of the flexible polyvinyl chloride (PVC) geomembrane by heating during the extrusion process. The geotextile is also designed to function as a drain to evacuate any water between the concrete and the geomembrane. The PVC and geotextile layers are 2.5 and 1.5 mm thick, respectively, and weigh 3,250 and 500 g/sq m, respectively. The geocomposite is manufactured in rolls with a minimum width of 2.05 m and lengths dependent upon the specific application. In dam applications, each roll is long enough to cover the height of the upstream face where it is to be installed, thus avoiding horizontal welds.

Installation

In early applications, geomembranes were attached directly to the upstream face of concrete dams with nails or adhesives. In more recent applications, geocomposites have been installed with stainless steel profiles anchored to the face of the dam. This system (SIBELON SYSTEMS DAMS/CSE) consists of two vertical U-shaped sections fabricated to fit one inside the other to form a continuous rib (Figure 1). In addition to a uniform, continuous anchorage of the geocomposite, the system also allows the geomembrane to be pretensioned, thus eliminating the problem of sagging caused by the weight of the geomembrane.

Profiles are not installed near vertical monolith joints to allow the elastic geocomposite to accommodate longitudinal joint movements. The vertical anchorage and tensioning profiles are fabricated in 1.4-m lengths with slots for the threaded rods to allow relative vertical displacements between adjacent monoliths.

Any water which might collect behind the geocomposite is conveyed, at atmospheric pressure, along the profiles to the heel of the dam where it can be collected in drainage pipes. A high-density polyethylene geonet, 4 mm thick with a diamond-shaped mesh, can be installed behind the geocomposite to increase the drainage capacity of the system. The perimeter of the membrane system is sealed against the concrete face with stainless steel profiles to prevent intrusion of reservoir water. A flat profile is used for the horizontal seal at the crest of the dam, and C-shaped profiles (Figure 2) are used along the heel of the dam and the abutments.

Applications

Geomembranes have been used to rehabilitate concrete, masonry and rockfill, gravity dams and concrete arch dams including multiple and double curvature arches. Geomembranes have also been used to rehabilitate reservoirs and canals and to provide a water retention barrier on the upstream face of new dams constructed with roller-compacted concrete.

Lake Baitone Dam

The first Italian application of a geomembrane for rehabilitation purposes was at the 37-m-high Lake Baitone Dam (Cazzuffi 1987). The upstream face of the stone masonry and cement mortar structure, completed in 1930, was lined with a series of vertical, semicircular concrete arches with an internal diameter of 1.7 m. Deterioration of the concrete

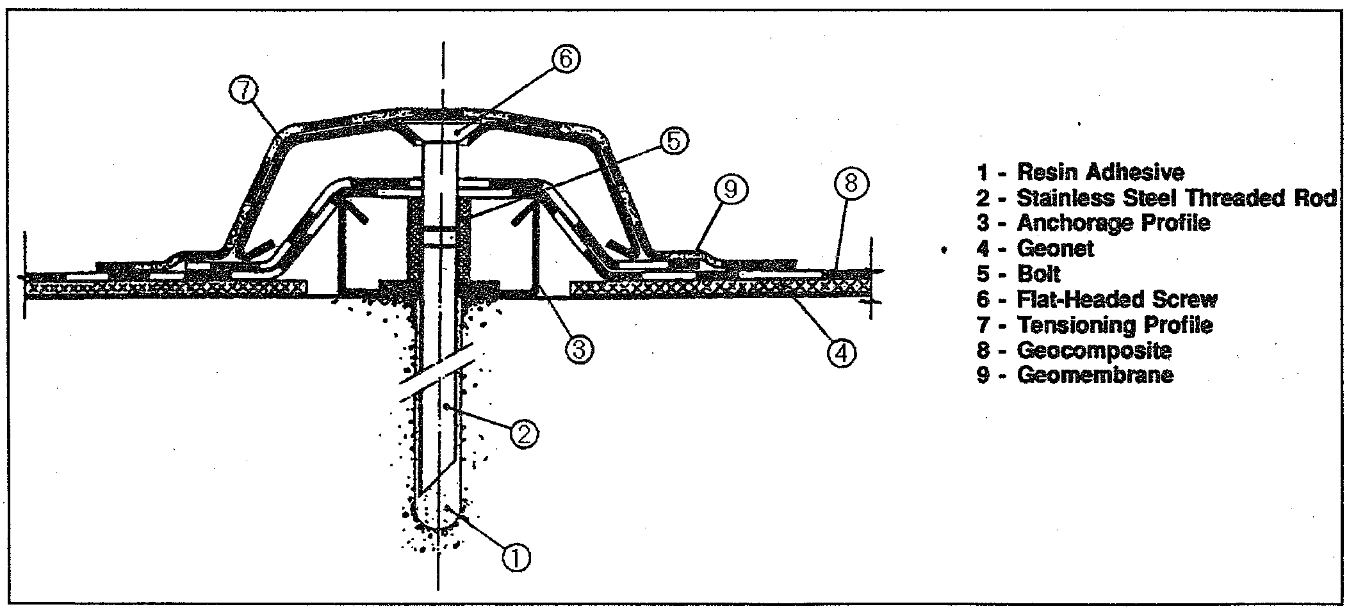


Figure 1. Detail of vertical anchorage and drainage profile

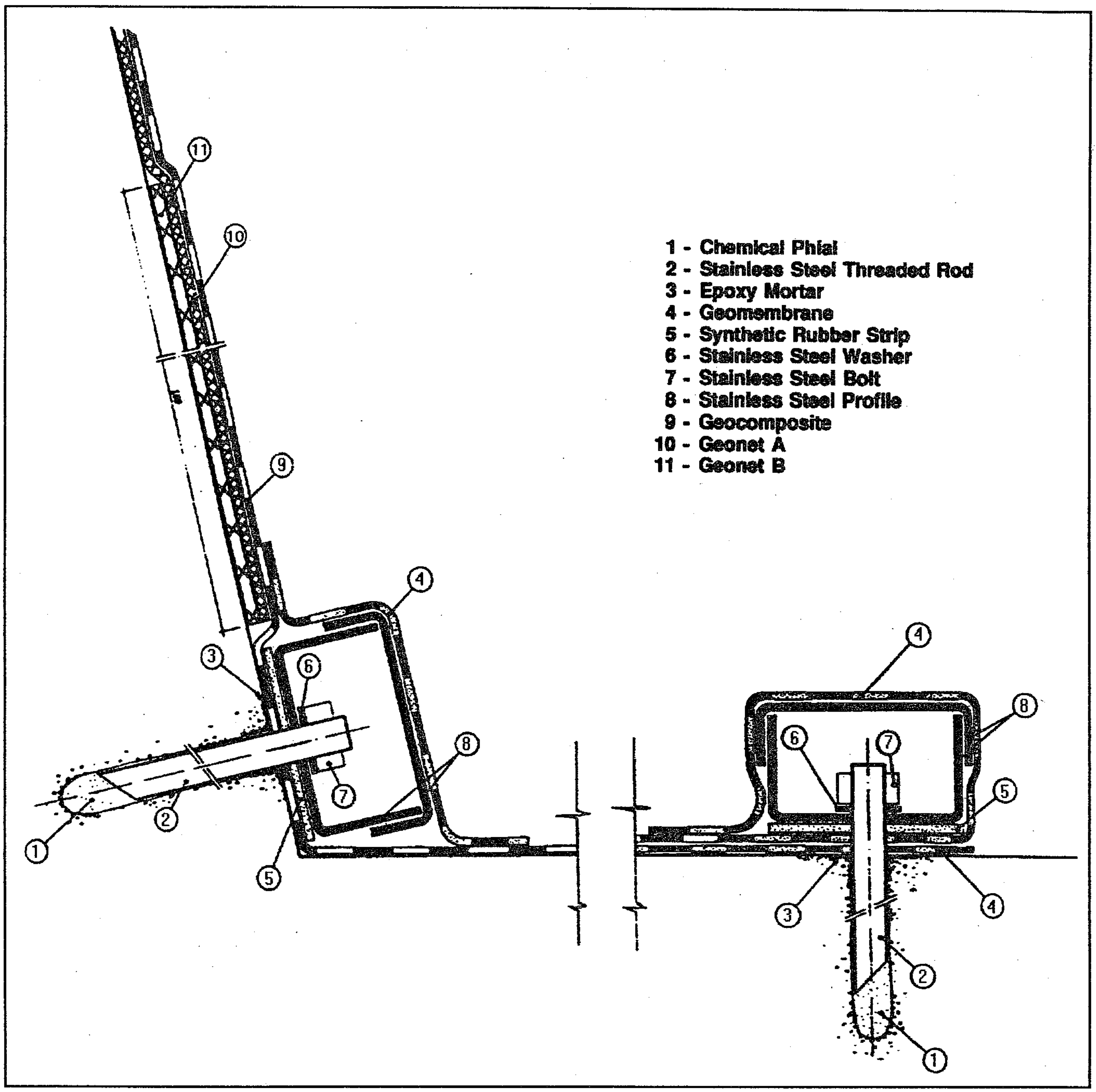


Figure 2. Detail of sealing profiles

surfaces required rehabilitation in 1970 with a 2-mm-thick polyisobutylene geomembrane applied directly on the arches with an adhesive, without any external protection.

After more than 20 years in service, the geomembrane exhibited good adhesion, and its performance was considered to be satisfactory, although there had been some damage caused by heavy ice formation which was quickly and easily repaired. However, when similar repairs were later made on concrete gravity dams, the results were clearly negative (Monari and Scuero 1991). This difference in performance was attributed to the extensive network of cracks in the thin arches which provided for natural drainage of vapor pressure in contrast to the limited drainage associated with the thicker gravity sections with minimal cracking. This experience and subsequent laboratory tests led to the conclusion that geomembrane repair systems should be mechanically anchored and must permit drainage of any water which might be present behind the geomembrane.

Lake Nero Dam

This 40-m-high concrete gravity dam with a crest length of 146 m was completed in 1929 near Bergamo, Italy. Over the years, various repairs, including grouting of the bedrock and shotcreting of the upstream face were conducted. However, these attempts to eliminate leakage through the dam and foundation and deterioration of the concrete caused by aggressive water and cycles of freezing and thawing proved to be temporary or inadequate.

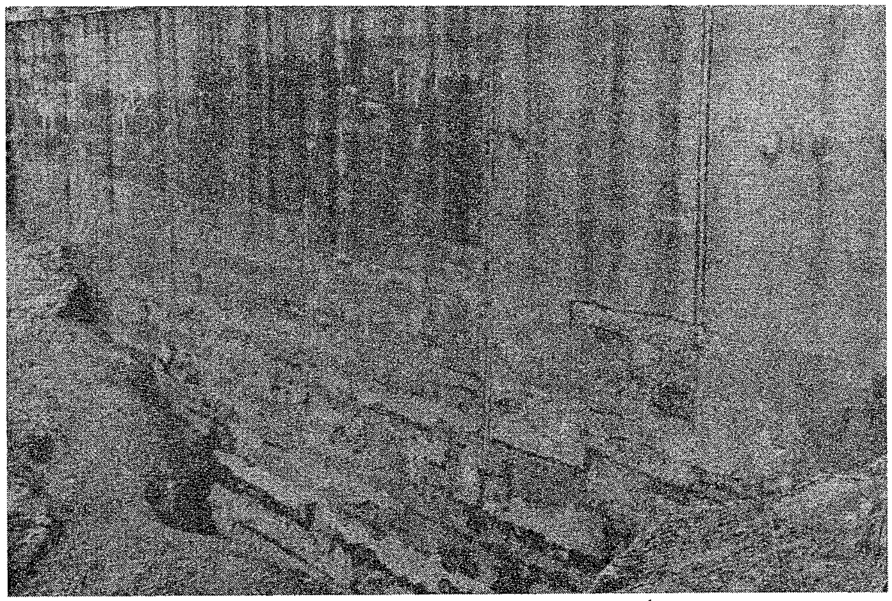
In 1980, a geocomposite was installed on the upstream face of the dam and anchored with steel profiles (Monari 1984). Self-hoisting platforms secured to the dam's crest were used in the installation (Figure 3), which required 90 days to complete. The geocomposite was unrolled from the top of the dam down to the base. Adjacent sheets were overlapped, and the vertical joints were welded prior to horizontal prestressing. The lower ends of the sheets were anchored to the base of the dam with metal plates.

Installation of the geocomposite reduced leakage from 50 L/sec to 0.27 L/sec with only 14 percent of the leakage through the geocomposite. After 10 years in service, the geocomposite had not required any maintenance, nor had the lining lost any of its original efficiency (Monari and Scuero 1991).

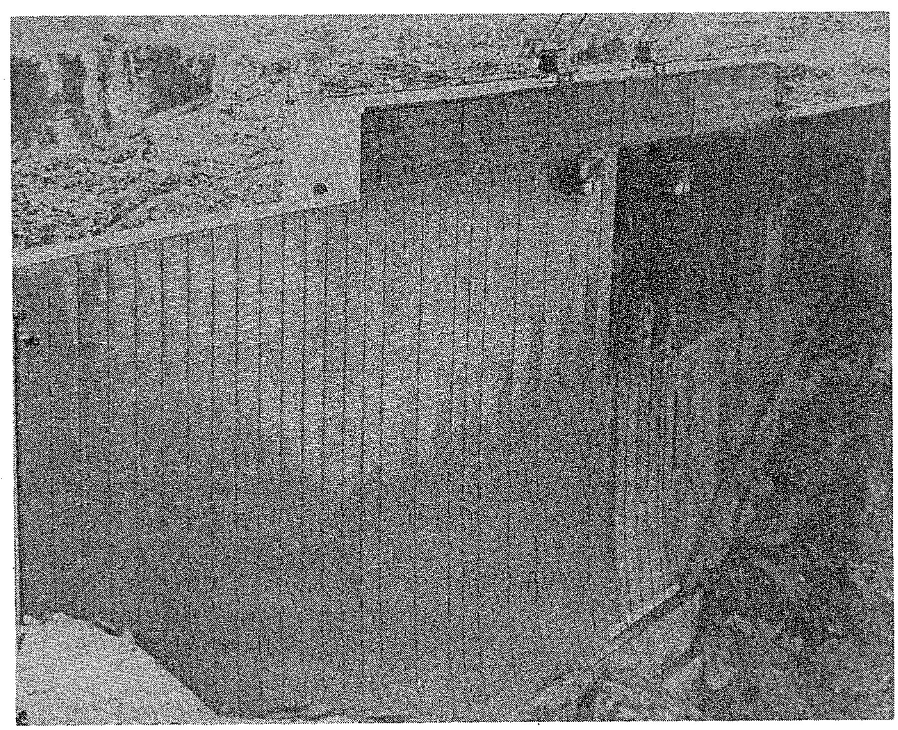
Cignana Dam

This 58-m-high concrete gravity dam with a crest length of 402 m was completed in 1928. Located at an elevation of 2,173 m above sea level, the concrete exhibited evidence of considerable freeze-thaw degradation. Also, the dam had major leakage problems despite extensive maintenance efforts, including application of paint-on resin membranes.

A 2.5-mm-thick geocomposite was used to rehabilitate the dam in 1987. The repair was similar to that of Lake Nero Dam except that the vertical steel profiles were embedded in a layer of reinforced shotcrete used to resurface the upstream face of the dam. After removal of deteriorated concrete, a layer of shotcrete was applied, vertical profiles were installed, and the surface between adjacent profiles was made level by



a. Prior to rehabilitation



b. During rehabilitation

Figure 3. Upstream face of Lake Nero Dam

filling with a second layer of shotcrete (Figure 4).

Pracana Dam

This 65-m-high concrete buttress dam with a crest length of 240 m is located on the Ocreza River in Portugal. Cracking occurred soon after construction, causing leakage at the downstream face. An attempt to reduce leakage by grouting the cracks did not result in a durable solution to the problem; therefore, a comprehensive rehabilitation was initiated in 1992. Application of a geocomposite to the upstream face of the dam was a major component of the rehabilitation.

A high-density polyethylene geonet was attached to the upstream face to increase the capacity of the geomembrane system to drain moisture from the concrete and any leakage through the geocomposite. An additional geonet was installed for a height of about 1 m above the heel of the dam to assist in conveying the water to drainage pipes. Ten near-horizontal holes were drilled from the heel of the dam to the downstream face to accommodate the drainage pipes. The geomembrane system was divided into six independent compartments from which water is piped to the downstream face for volumetric measurements. This design provides improved monitoring of the drainage behind the geomembrane.

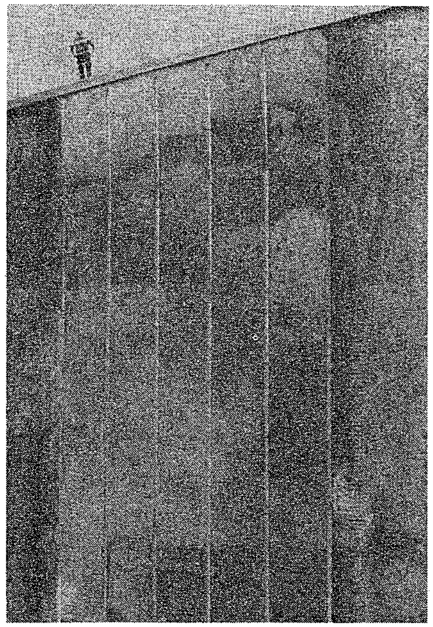


Figure 4. Steel profiles embedded in shotcrete, Cignana Dam

Approximately 8,000 sq m of geocomposite were installed

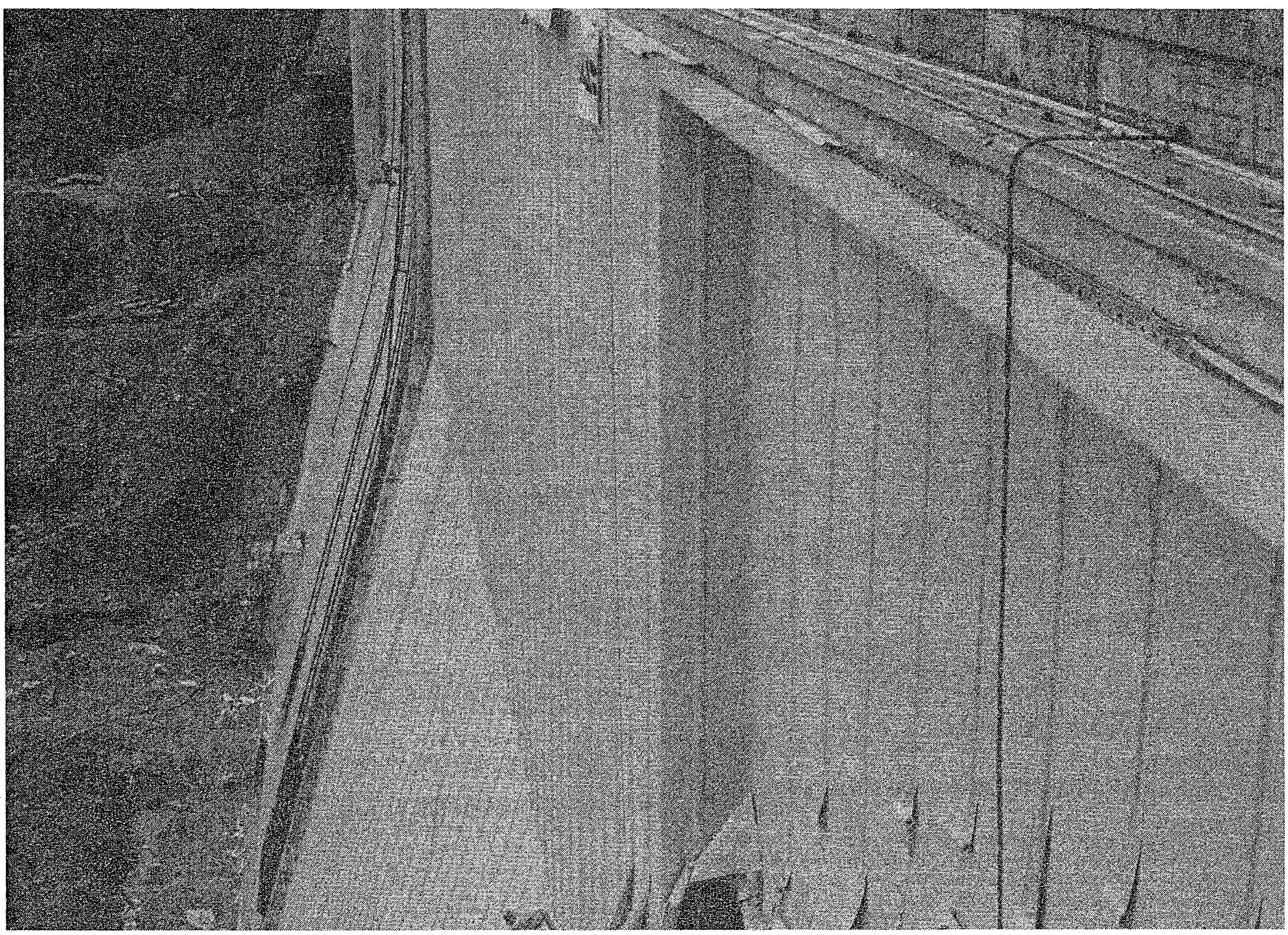


Figure 5. Installation of geocomposite, Pracana Dam

with steel profiles shown in Figures 1 and 2. Vertical joints between rolls of the geocomposite and geomembrane strips covering the profiles were welded to provide a continuous water barrier on the upstream face of the dam. A hot air jet was used to melt the plastic, and the two surfaces were then welded together by hand pressure on a small roller.

A wire system was installed behind the geocomposite to aid in future monitoring and maintenance of the geomembrane. A sensor sliding on the surface of the geomembrane can detect any anomalies in the electrical field thus locating any discontinuities in the geomembrane.

Conclusions

Geomembranes and geocomposites have been installed on the upstream face of more than 20 old concrete dams during the past 23 years. The success of these systems in controlling leakage and arresting concrete deterioration, and the demonstrated durability of these materials are such that these systems are considered competitive with other repair alternatives.

With a few exceptions, geomembrane installations to date have been accomplished in a dry environment by dewatering the structure on which the geomembrane is to be installed. Dewatering, however, can be extremely expensive and in many cases may not be possible because of project constraints.

A geomembrane system that could be installed underwater would have significantly increased potential in repair of hydraulic structures. Consequently, research has been initiated to develop a procedure for underwater installation of geomembrane repair systems. The objectives of this REMR research are to (a) develop concepts for geomembrane systems that can be installed underwater to minimize or eliminate water intrusion through cracked or deteriorated concrete and defective joints, and (b) demonstrate the constructibility of selected concepts on dams and intake towers.

For additional information, contact James E. McDonald at (601) 634-3230.

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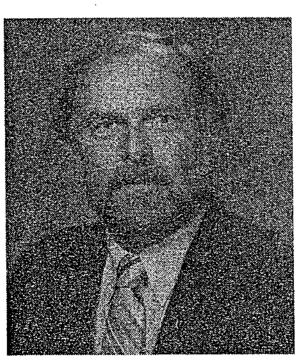

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Toe stability in a combined wave and flow environment

by
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Background

The design and construction of a stable toe for a rubble-mound structure are as important to a repair program as are the design and repair of primary armor slopes. In many cases, an unstable toe will result in failure of an otherwise adequately repaired structure. A survey conducted by the Coastal Engineering Research Center (CERC), Waterways Experiment Station (WES), found that toe instability was evident on the coastlines of the Atlantic, Pacific, Gulf of Mexico, and Great Lakes (Markle 1986). All sites reporting toe instability were located in an environment in which both waves and flow were present. Toe stone is difficult to replace; therefore, it is important to determine the stablest and most economical size for design and repair.

Research is being conducted under the REMR Research Program to develop a method, through physical model tests, for determining the size and placement of toe stone necessary to achieve a stable toe subjected to waves and flows. To date, a fixed/movable bed test section with a circulation system has been constructed in CERC's L-shape flume, and testing has been initiated. The circulation system was configured so that both flood and ebb flows up to 2 ft/sec (approximately 10 ft/sec, prototype) could be generated. The test section was positioned on a flat bathymetry preceded by compound approach slopes.

Tests have been previously conducted with a fixed-bed physical model to evaluate the stability of rubble-mound structure toes exposed to a breaking wave environment (Markle 1989). These tests did not address the combined effects of waves with river- or tidal-induced flow. The outcome of these tests provided design guidance for sizing toe stone to be placed during the repair and/or rehabilitation of rubble-mound structure heads and trunks exposed only to breaking waves. Markle developed a relationship between toe stability for depth-limited breaking waves and the depth ratio at the toe. The relationship is shown in Figure 1 with design curves developed previously by Brebner and Donnelly (1962) for vertical breakwaters. The stability number, N_s, is directly proportional to the design wave height, H_d, and inversely proportional to the toe berm weight. For a given H_d and depth ratio, the proper toe berm size can be determined from Figure 1. An important discovery noted by Markle is that stability numbers are much lower for breaking waves on rubble-mound breakwaters than the curves developed by Brebner and Donnelly. This means that heavier toe sizes are required than were previously used.

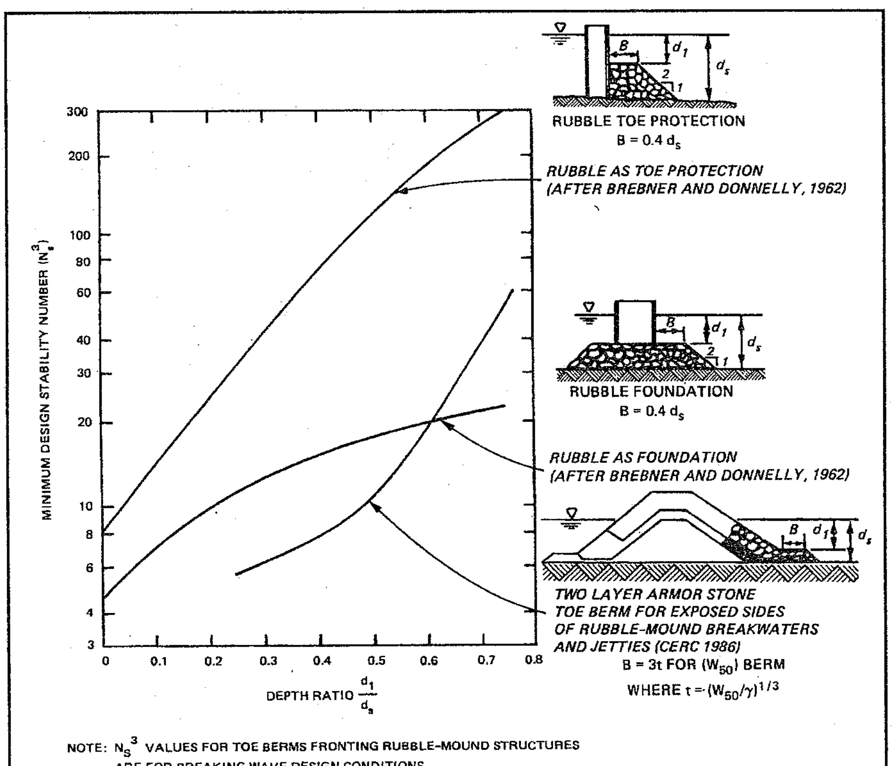

Figure 1. Minimum design N_s^3 for rubble-mound structures

Present work

The test area consists of the test jetty, modeled on the order of 1:25 model to prototype, and a companion jetty. The test jetty is oriented at a 70-deg angle-to-wave attack (Figure 2). The companion jetty was placed near the test jetty to reproduce reflections common to twin jetty inlets and also to assist in directing current flow. Ebb flow currents are introduced at the manifold (Figure 2).

The test jetty, which is 16 ft long, is composed of two sections. The shoreward section is constructed of plywood with stones sealed onto the face. The surface was found to reproduce reflections similar to that of a rubble-mound structure. Because it does not require rebuilding and is easy to transport to other parts of the model, it is convenient to use in place of a rubble-mound structure. The test, or seaward, section was built as a rubble-mound structure using dolosse as an armor unit. It was desired to build a section of highly stable armor units so that toe instability would be isolated. Toe stone was placed around the test section in the manner recommended by Markle (1989), i.e., three stones wide and two stones high. For maximum testing efficiency, seven toe stone cells were used around the perimeter of the test section. Each cell contained a different stone size from the adjoining sections. A schematic of the test section is shown in Figure 3. The stones were color coded by size to readily identify the location of damage. The purpose of using varying stone sizes around the structure was to locate areas vulnerable to wave and flow. Toe failure may occur for certain wave and flow conditions in a section containing a heavier stone, whereas a lighter stone may be stable in a different section because of the interaction of the structure position and shape with the waves and currents. A photograph of the test section is shown in Figure 4.

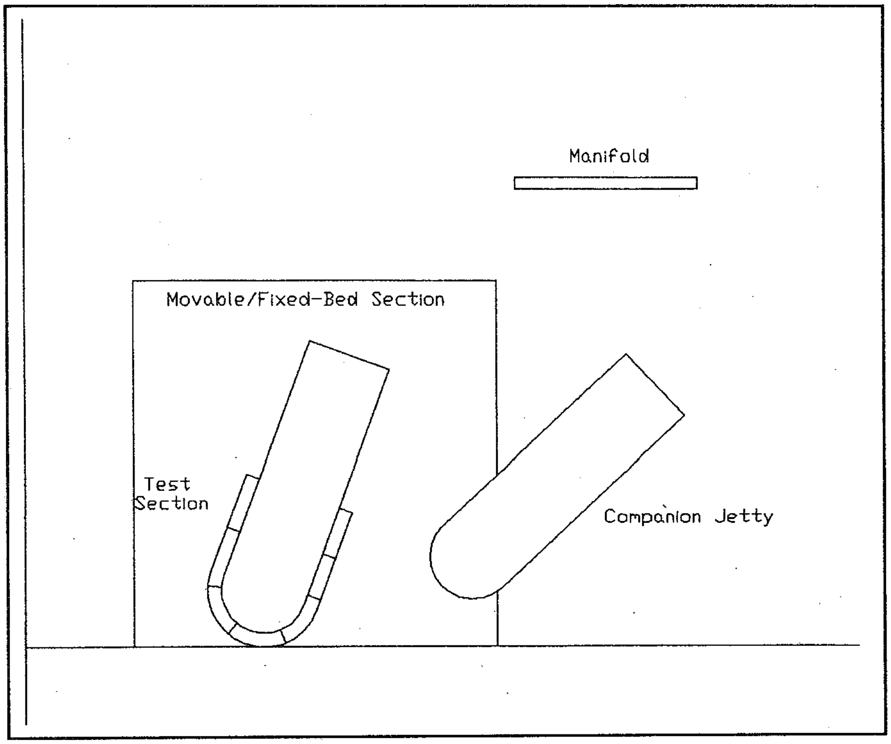

Figure 2. Layout of toe stability test section

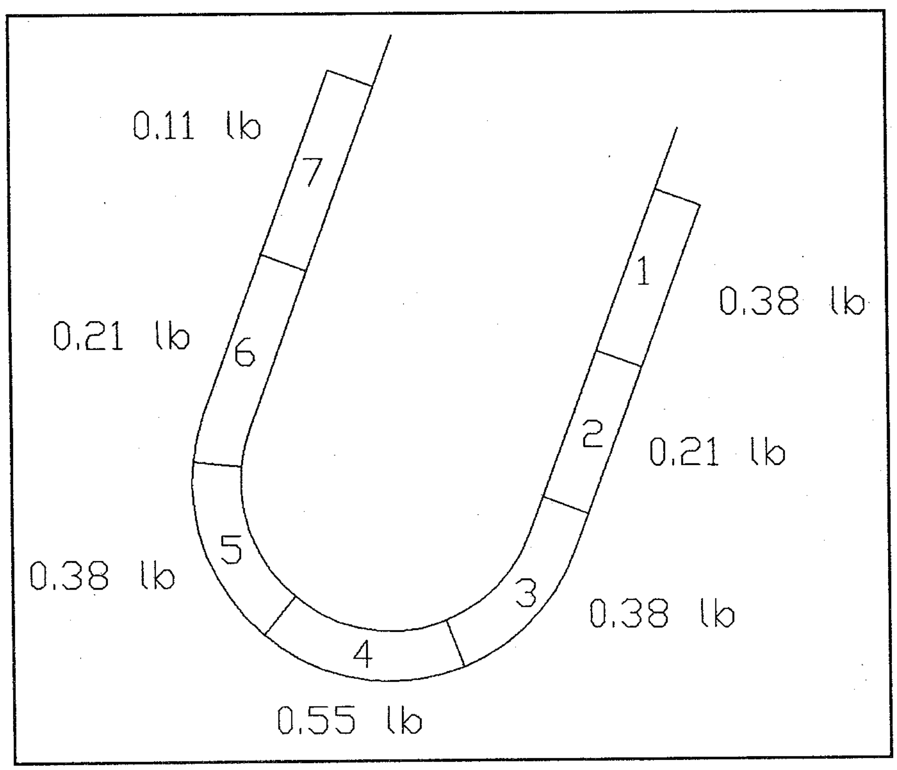

Figure 3. Location of berm stone sizes in test section cells

Both qualitative and quantitative results were taken during tests. Results indicated that for a given ebb-flow condition, a wave condition existed that would cause a pulsating effect at the toe berm. This effect was a result of the waves breaking and running up the structure and then running down. While waves ran up the structure, the water surface elevation increased, orbital current velocities of waves opposed the ebb-current velocities, and net current flow was at a minimum. However, during rundown, the water surface elevation decreased and wave orbital velocities and ebb-current velocities were in phase. For this condition, the net velocity was at a maximum, and the water surface elevation was at a minimum, which reduced the flow volume, and by continuity, velocity increased.

Quantitative results were obtained by determining the damage to each cell. Cells were not considered damaged if 2 percent or less of the total number of stones in the cell were displaced. The stability of each section was determined using the stability number, N_s. The stability number is a function of the design wave height, H_d, which is the wave height that causes 2-percent damage to the section (2 percent of the number of stones placed in that particular toe section), armor unit weight, armor unit density, and specific gravity of the individual armor unit relative to the water in which it is placed.

Toe stability tests were initiated for regular (monochromatic) waves at a 1.0-ft water depth in the model and ebb flows of 0.0, 1.0, and 1.5 ft/sec between the jetties. The test section was subjected to waves ranging from 0.1 to 0.8 ft,

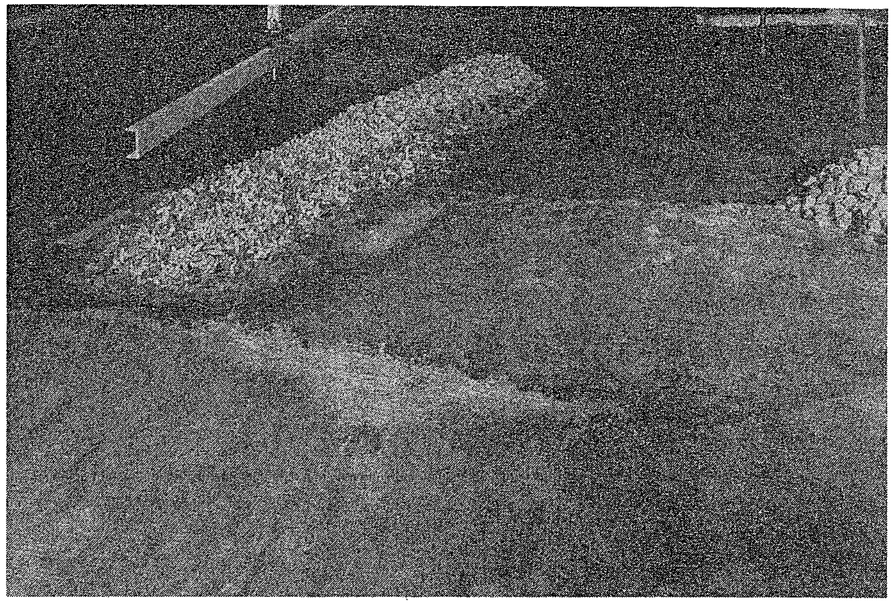

Figure 4. Toe stability tests

beginning with the lower wave height. After damage was noted for the lower wave height, the generator stroke was increased incrementally up to and greater than the stroke necessary to produce breaking waves. To minimize oscillations in the basin, waves were generated in five bursts of 3 min each per wave height. The number of berm stones displaced was recorded after each wave burst, percent damage was determined, and H_d was calculated for each berm cell. A comparison of H_d with magnitude of flow for one wave period is shown for each berm cell in Figure 5. Because design wave height is the height which causes 2-percent damage, it is inversely proportional to damage of a structure; therefore, Figure 5 also gives a comparison of percent damage to each cell. The figure shows that the damaging wave height to the section, i. e., less damage, is highest for nonflow conditions, slightly less for flows of 1.0 ft/sec, and lowest for 1.5-ft/sec flows. Design wave height was near constant for Cells 5, 6, and 7 (the outer cells) because the current did not flank the cells and most of the wave energy had dissipated seaward of those cells. The decrease in H_d with ebb flow for Cells 1 through 4 is caused by the current modifying the incident wave shape. As waves approach the test section and encounter an opposing current, they become steeper and break at lower wave heights than waves with no flow present. Therefore, waves which normally would not damage a structure could cause damage if a flow was present.

Stability number cubed versus the relative depth is shown in Figure 6. The nonlinear shape of the points are due to the placement of different size stone around the test section. As more data are analyzed for lower water depths and stone sizes are placed in different cells around the structure, a curve following the trend can be drawn. Stability numbers for no flow and 0.5-ft/sec conditions are on or above the curve of minimum values obtained by Markle (1989). Conditions with a 1.0-ft/sec flow fall slightly below the curve, and conditions

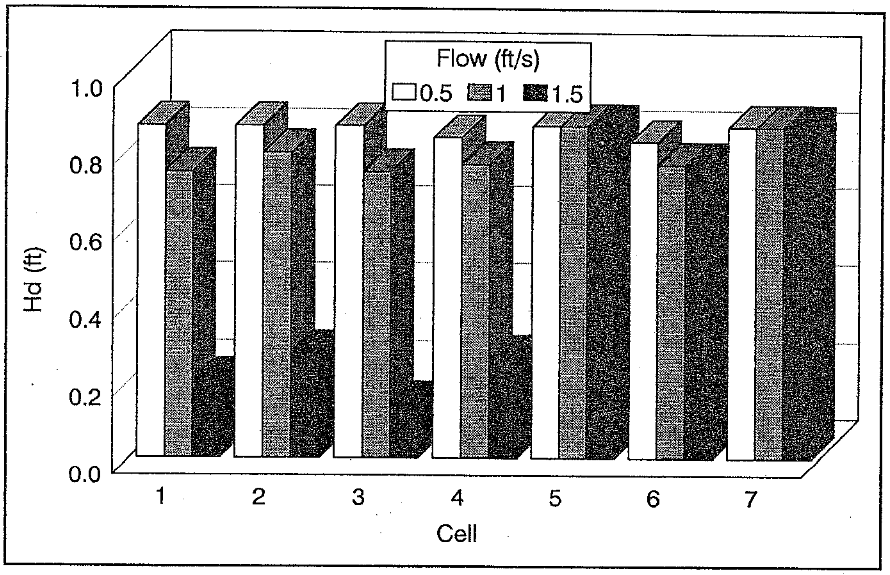

Figure 5. Comparison of design wave height with flow magnitude for each test cell

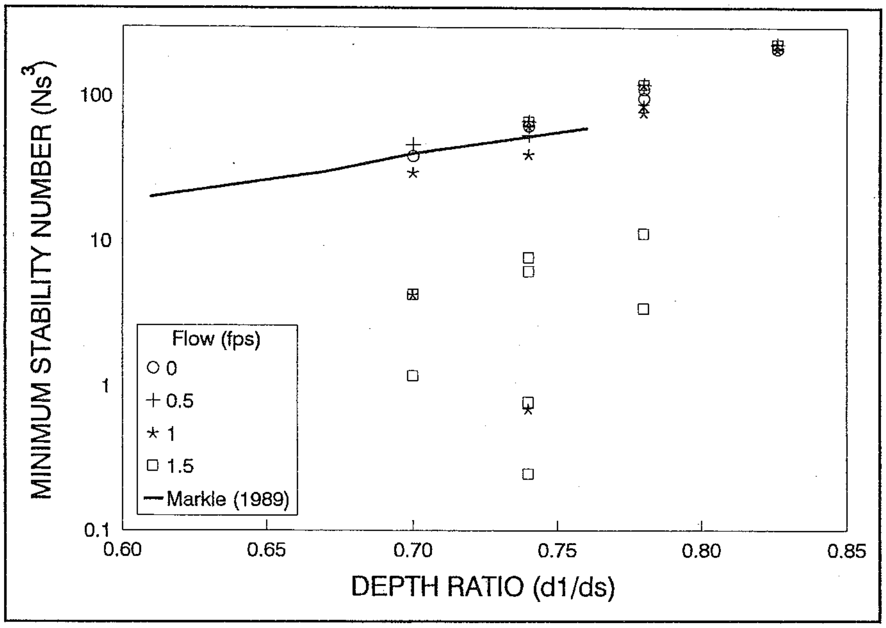

Figure 6. Stability number cubed versus relative depth

with a 1.5-ft/sec flow are significantly lower than the no-flow values. The flows used for the test conditions represent prototype currents between 5.0 and 7.5 ft/sec, which are not uncommon to many inlets in the United States. Much larger toe stone is required in areas that are subjected to both wave and flow conditions.

Preliminary results indicate that toe berms are less stable in environments in which ebb flows exist. Damage may occur to the toe berm if ebb flows are present for offshore wave heights much lower than waves in a no-flow condition. The tests show that ebb flow should be considered for toe repair and rehabilitation design.

Future work

Toe stability tests will continue for regular and random waves with ebb- flow conditions. Because damage to the toe increased dramatically between tests with flows of 1.0 and 1.5 ft/sec, additional tests with a 1.25-ft/sec flow will be conducted. Results will be compared with the findings of Markle (1989), and guidance for toe berm sizing and placement will be developed for ebb-flow conditions.

From tests and field observations, toe failure and scour appear to occur where waves and flows combine to create high velocities. In addition to toe stability tests being conducted, two- and three-dimensional research tests are being performed under the Coastal Research Program. The purpose of the tests is to determine the location of maximum bottom velocity of waves impinging upon rubble-mound structures by obtaining the phase shift of the reflected wave. Hughes (1992) derived a predictive equation for root-mean-square velocity bottom velocity and found good results when the equation was applied in cases with a perfectly reflecting vertical wall, i.e., $\cos\theta = 1$, in which θ is the phase shift due to reflection. The difficulty in using the equation for rubble-mound structures or any other sloping structure is determining $\cos\theta$ if it is not equal to one.

To investigate the phase shift further, basic two-dimensional tests were conducted with a smooth sloping structure installed. Reflection measurements for a variety of slopes were obtained over a range of incident wave periods and heights. From these measurements, a predictive equation for phase shift due to reflection was developed for given incident wave conditions and structure slope. Using this equation, the calculated phase shift can then be used in the method of Hughes (1992) to predict the location and magnitude of bottom velocity, but only for normally incident waves. To expand on the two-dimensional results, a newly developed acoustic current meter will be used in the three-dimensional basin to obtain reliable velocity measurements for conditions in which wave approach is oblique. Results of these tests will help predict the location along the structure in which velocities are maximum and likewise have the highest probability of toe failure. These tests will supplement the ongoing toe stability tests, which will provide guidelines for proper toe armor size and berm width design for wave and flow conditions.

For additional information, contact Ernest R. Smith at (601) 634-4030.

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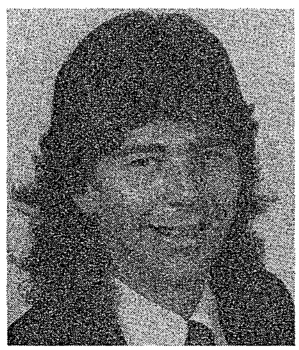

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Changes in REMR-II key personnel

Dave Wingerd, Hydraulics and Hydrology Branch, Civil Works, has been appointed Technical Monitor of the Hydraulics Problem Area. W. Milton Myers, Soil Mechanics Branch, Geotechnical Laboratory, WES, is the new REMR-II Geotechnical-Soils Problem Area Leader, replacing Gene P. Hale, who recently retired from the Corps of Engineers. Roy Deda has been named the Field Review Group Operations Member from the North Central Division. Bob Neal, who formerly held this position, retired from the Corps of Engineers last year.

We extend our appreciation to Gene and Bob for their contributions to the REMR-II Research Program and wish them well in their future endeavors.

Heated water jet for melting ice

by
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Ice accumulations at locks and dams can interrupt the normal operation of locks, make tainter gates inoperable, and create unsafe working conditions for personnel. A survey made by the Cold Regions Research and Engineering Laboratory (CRREL) (Haynes et al. 1993) determined that many locks and dams have severe ice problems during the winter months. The responses to the survey also indicated that there are no known solutions to a number of these problems and that manual solutions, such as ice chipping, are still being widely used.

To lessen reliance on manual solutions, the concept of using a water flow inducer has been investigated at CRREL. A flow inducer (also called a flow developer) is a submersible electric motor with a propeller mounted on its shaft. The propeller induces a flow of water that can be directed at ice to melt it. Laboratory tests and analyses (Ashton 1989) and examination of the icing problems around locks and dams (Haynes et al. 1993) indicate that many ice problems can be reduced or eliminated by a flow inducer.

System design

In addition to using a flow inducer to melt ice, the CRREL research team decided to add heat to the water flow from the inducer. A hollow aluminum pipe housing was mounted in the flow. Three 500-W cartridge heaters were placed inside the pipe wall at 120-deg intervals. The cartridge heaters were 3/8 in. in diameter and 3 in. long, with waterproof lead wires. All electrical connections were waterproofed. The electric motor of the flow inducer operated at 913 W. This power was eventually dissipated as heat was added to the water. Figures 1 and 2 show the system.

Only three 500-W heaters were installed in the housing so that the system could operate on a 110-VAC, 20-amp circuit. Additional heaters could be installed to increase the heat added to the flow. However, a larger capacity circuit would then have to be used.

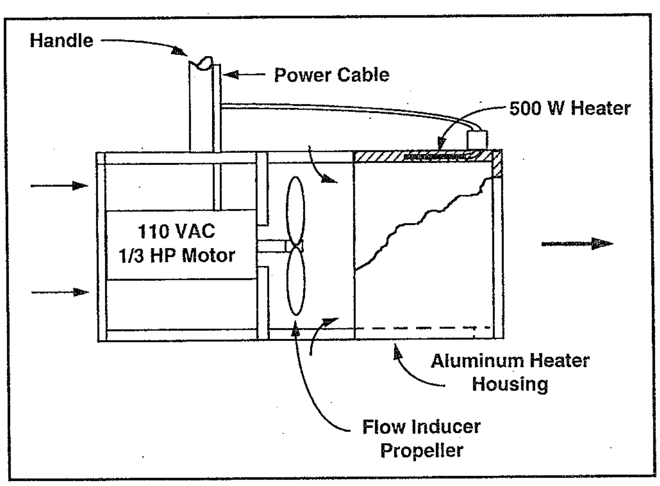

Figure 1. Schematic of portable heated water jet system.

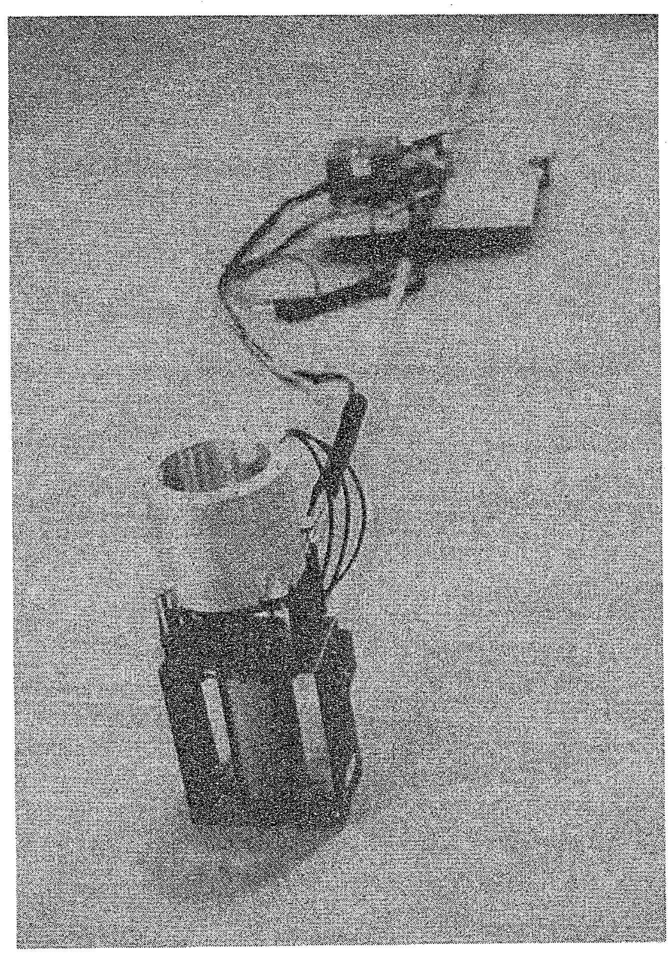

Figure 2. Heated water jet system with three 500-W heaters in the aluminum housing

During final assembly, the housing was bolted onto the flow inducer. Also, a handle (made of 1-in. conduit pipe) and electrical switches were attached to a plate for positioning the flow inducer in laboratory tests.

Laboratory tests

The test basin in CRREL's Ice Engineering Facility was used for the tests. This basin is 8 ft deep, 30 ft wide, and 110 ft long. The basin room is refrigerated and can typically grow 1 in. of ice in 16 hr at an air temperature of 5° F.

Tests were designed to melt ice along the concrete walls of the basin. Prior to testing, a 3.5-in.-thick ice sheet was grown in the basin. The ice at the ice-concrete interface was 12 in. thick because of the higher thermal conductivity of the reinforced concrete and a solid heat flow path to the cold ambient air. These factors produce lower temperatures in the concrete than in the ice below the water level and cause the ice to thicken. This thickening, or ice collar, is observed in the field on concrete walls and steel tainter gates. The tests therefore simulated a real field condition, as illustrated in Figure 3.

To conduct the test, a hole was cut in the ice sheet next to the concrete wall, as shown in Figure 4. The heated water jet system was then lowered into the hole 1 ft below the water level

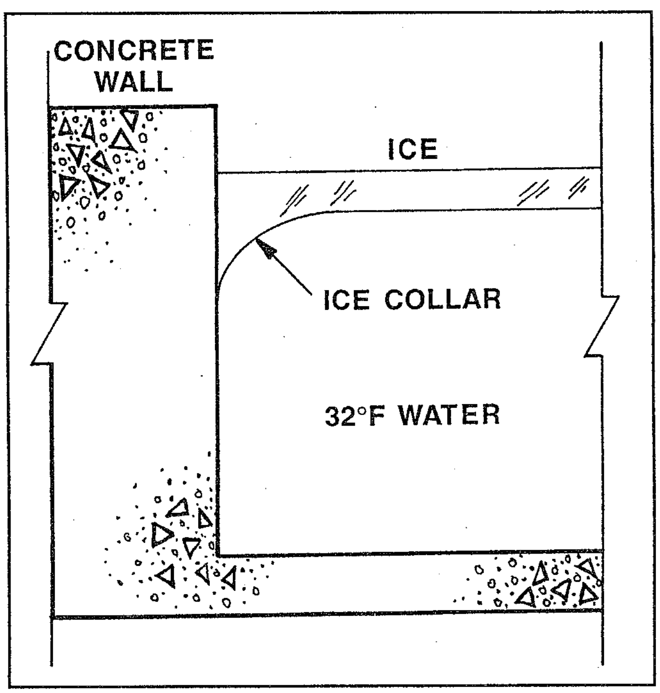

Figure 3. Drawing of an ice collar grown on a concrete wall

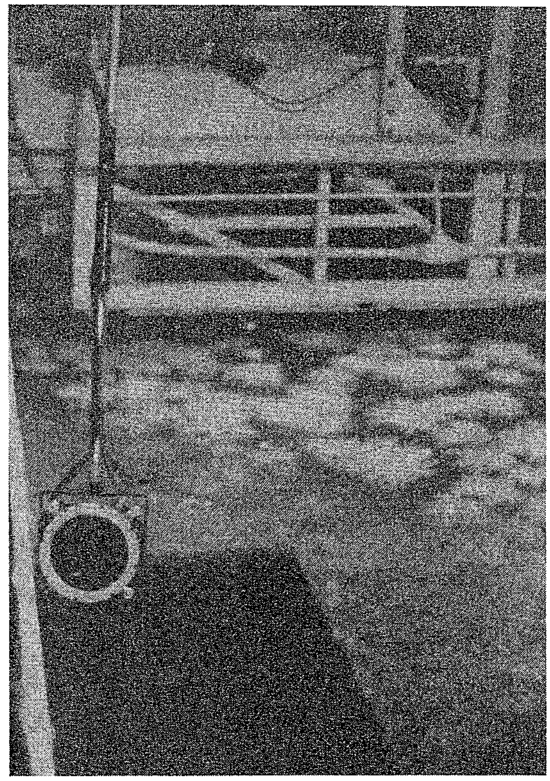

Figure 4. Heated water jet system being lowered into a hole in the ice

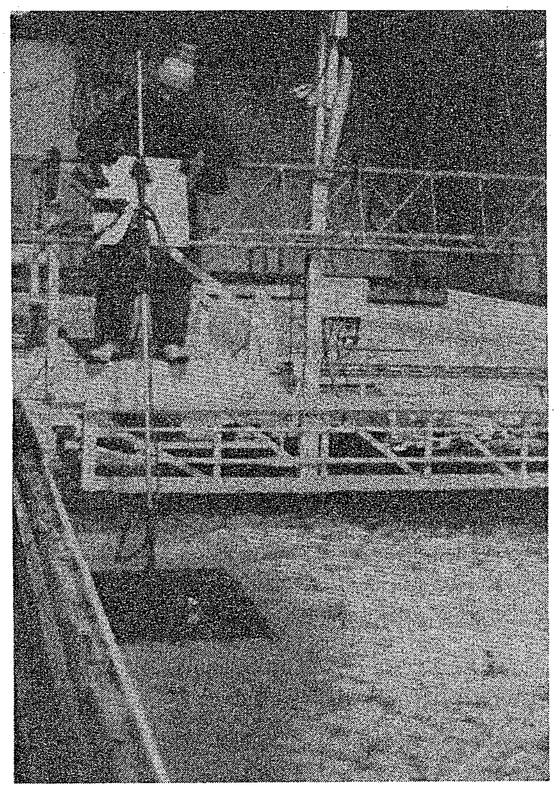

Figure 5. Heated water jet lowered in position on melt ice

(Figure 5). The flow inducer was turned on first; then, for the tests with heat, the three 500-W heaters were turned on. Approximately every hour, measurements were made of the melting ice along the wall. Typical results of a test are given in Figure 6.

The advantage of the heated water system decreases with time. For example, after 2 hr the part of the ice collar melted with heat was about 2.2 times the distance of that portion melted without heat. After 4 hr, the distance melted with heat was only 1.8 times that melted without heat. This is partly due to the relationship between flow momentum, distance, and heat dissipation produced by the particular flow inducer tested. A larger or smaller flow inducer would produce different results.

Field applications

There are many ice problem areas at locks and dams. Some of these are very local, such as ice in a miter gate quoin or on a tainter gate seal plate. Some of these are larger, such as an ice collar on a lock wall or tainter gate. One of the advantages of the heated water jet system is that it is portable. It can be used in a variety of problem areas. Figure 7 illustrates the use of the system on an ice collar. It is also light and can be mounted on a railing or a gate structural member with C-clamps. For example, it could be mounted on a railing next to a miter gate quoin area for a few hours to remove the ice which might restrict the movement of the miter gate. In such a situation, it could be left unattended.

This system could be made with a range of flow-inducer sizes, heater sizes, and power requirements. Along with its versatility and portability, it offers an effective alternative to the manual ice chipping currently being used at many locks and dams.

For additional information, contact F. Donald Haynes (603) 646-4184.

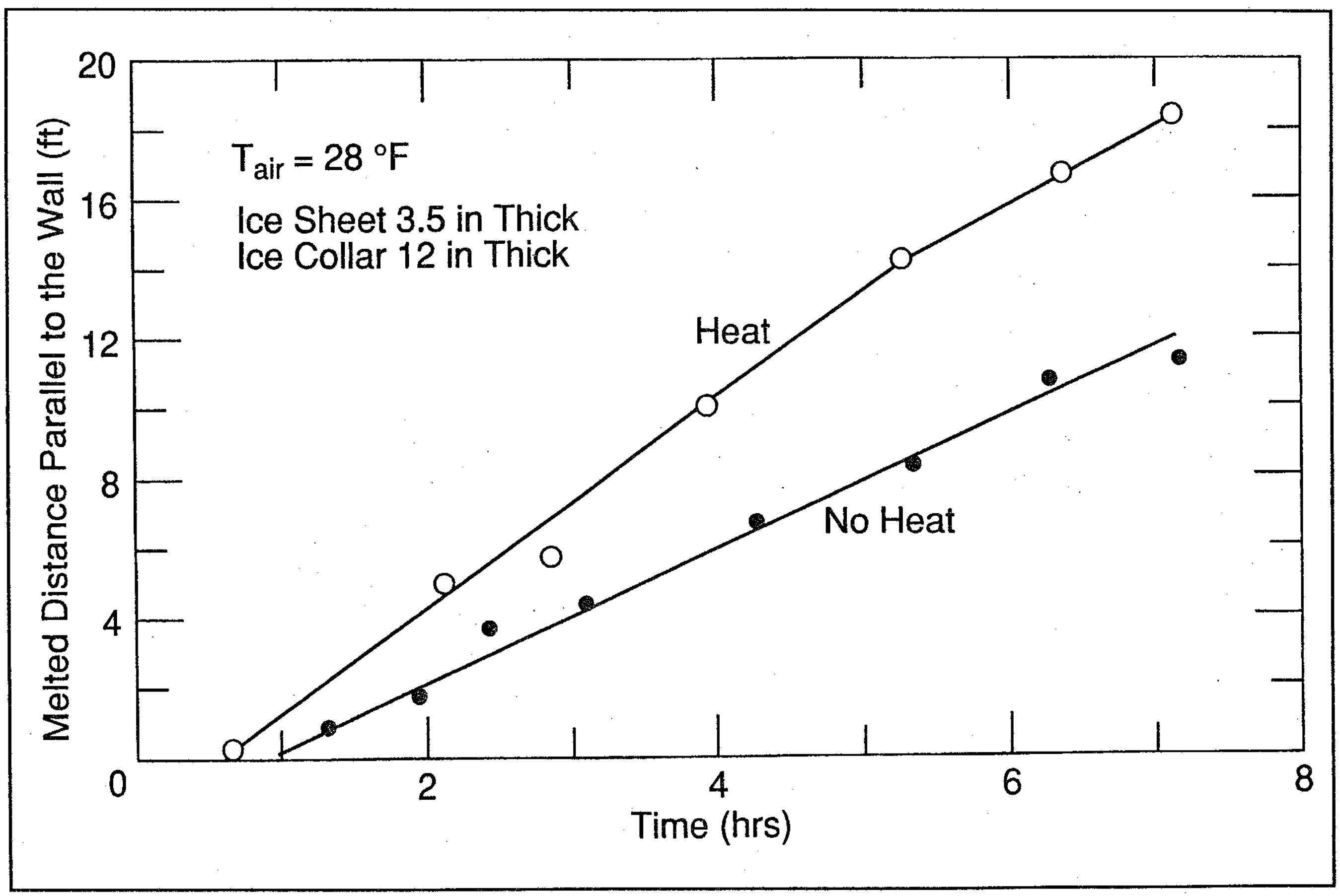

Figure 6. Typical test results showing the ice melted as a function of time

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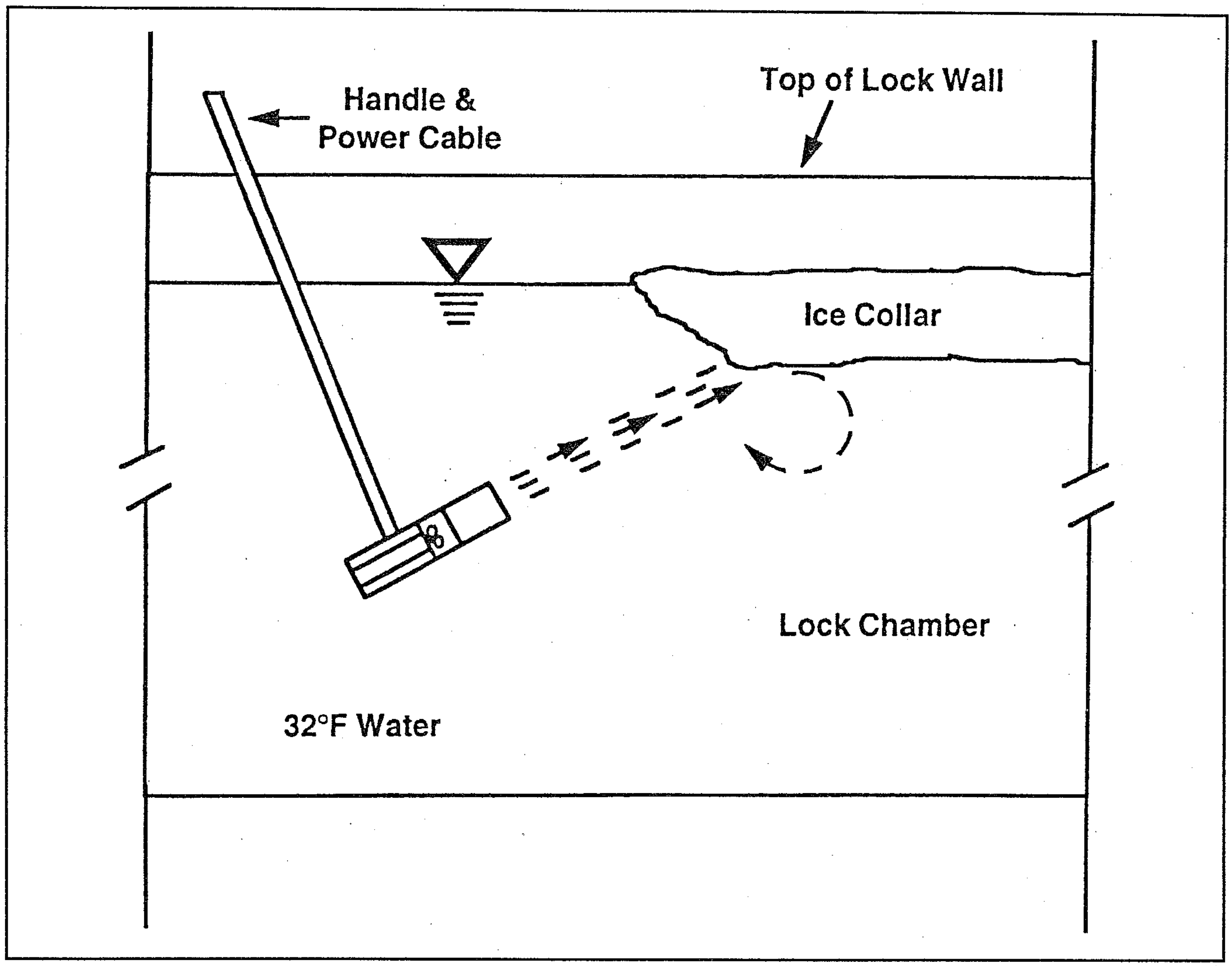
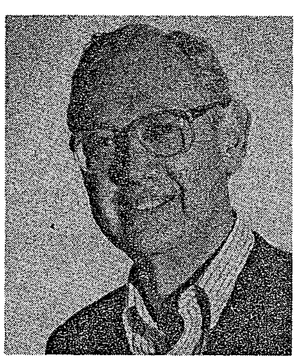


Figure 7. Sketch of the system in a field application melting an ice collar on a lock wall



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